SUPERSEDES MEMO TO DESIGNERS 3-1 DATED DECEMBER 1998

3-1 DEEP FOUNDATIONS

Deep foundations are structural assemblies that transfer load down through weak soil strata and into deeper and stronger strata to minimize the settlement of a structure. Caltrans deep foundations consist of a single pile or a group of piles with a pile cap. These deep foundation piles can be driven, drilled, cast-in-place, or alternatively grouted-in-place.

The Division of Structure Design (DSD) is responsible for calculating the pile load demands and for providing structure details. The Division of Structural Foundations (DSF) is responsible for providing foundation recommendations that include pile type and tip elevations (based on the load demands provided by DSD), construction recommendations (pile acceptance criteria, testing requirements, etc.), the Log of Test Borings, and Information Handouts. DSD and DSF will reach a consensus on pile type and special construction requirements. DSD is responsible for ensuring that the intent of the geotechnical and structural design is preserved in the contract plans and specifications. At the submittal of P&Q, any information absent from the Foundation Recommendations should be included in the project engineer's Memo to Specifications Engineer. Near PS&E, the Foundation Review meeting completes the process, allowing DSF commentary on the plans and specifications while in the presence of the specifications engineer and DSD's project engineer.

Current design practice in DSD specifies that abutments are designed by Working Stress Design (WSD) and bents/piers are designed by Load Factor Design (LFD). LFD is used at the bents because we can confidently estimate the maximum load on the foundation (i.e., column plastic hinge loads), but there is less confidence in the abutment loads, especially in the dynamic soil force. The structure designer needs to give the geotechnical designer the pile design demands for the applicable design procedures so that the geotechnical capacity of the pile selected will meet or exceed these demands. These loads are shown in the PILE DATA table on the contract plans. See Attachment 1 for various examples of PILE DATA tables.

Standard Plan Piles

The *Standard Plans*, Sheets B2-3 (400 mm CAST-IN-DRILLED-HOLE CONCRETE PILE), B2-5 (PILE DETAILS CLASS 400 AND CLASS 625), B2-6 (PILE DETAILS CLASS 400C AND CLASS 625C), and B2-8 (PILE DETAILS CLASS 900 AND CLASS 900C) include the upper limit of structural pile design capacities in tension and compression.

When a Standard Plan pile is specified, the contractor has the option of using any of the alternatives for that Class of pile. Should any of the Class piles be infeasible to construct, that alternative should be disallowed in the contract Special Provisions.

Special Consideration for Alternative 'X' Piles

The 305 mm square precast prestressed Class 400 and Class 625 concrete pile, Alternative 'X', does not have the lateral capacity necessary for the various pile spacing design charts in Section 6 of the *Bridge Design Details* manual for either Strutted Abutments, Cantilever Abutments, Type 1 Retaining Walls, or Counterfort Retaining Walls. If these design charts are used, the Special Provisions shall stipulate that Alternative 'X' piles must have a dimension 'T' not less than 350 mm for the specific locations involved. This information should be included in the Memo to Specifications Engineer.

Lateral Resistance

The allowable lateral resistance of a Standard Plan pile fully embedded in soil with a standard penetration resistance value, N, of 10 or greater and a 6 mm maximum horizontal deflection under Service Load is given in Article 4.5.6.5.1 of the *BDS (Bridge Design Specifications, AASHTO 16th Edition, 1996)*. The lateral pile resistance in *BDS* is based on soil failure and can be increased provided a geotechnical analysis, such as COM624 computer analysis, supports the increase. When the soil in the upper zone of the embedded piles has a standard penetration resistance value less than 10, the lateral resistance values are not applicable and a special pile design will be required. In the case of battered piles, the horizontal component of the axial load can be (two words) added to the lateral resistance. In all cases where the Standard Plans are used, the pile-to-pile cap connection is intended to be a pinned connection.

Driven Piles

Driven piles can be precast prestressed concrete, cast-in-steel-shell concrete, steel HP, steel pipe or timber. Piles with a solid cross section that displace the soil around the pile are displacement piles. Open cross sections, such as steel HP piles and open ended pipe piles, will either displace the soil or cut through the soil (non-displacement) depending on the properties of the soil and diameter of the pile. Typically, steel HP piles and open-ended pipe piles 600 mm and greater diameter are non-displacement piles. Such piles are useful for penetration where boulders or hard strata are expected.

Site specific issues including noise, vibration, ground heave, headroom, constructibility, and driveability must be considered when selecting driven piles. If liquefaction or scour potential exists, driveability must be evaluated to verify that the piles can penetrate to the required tip elevation.

To increase lateral capacity, driven piles may be battered. Typical Caltrans designs use a batter of 3:1. Where battered piles encroach on property outside of the right-of-way lines, the District Project Engineer should be informed that an easement is required.

DSF will recommend the preferred acceptance criteria for driven piles. For most applications, Standard Specification 49-1.08 (the ENR formula) is sufficient. More rigorous methods, such as the Wave Equation, may be specified for high-capacity piles. Piles with nominal resistance 900 kN and greater typically require the Wave Equation for acceptance.

Timber Piles

Timber piles can be specified where conditions are suitable, usually for temporary construction (e.g. railroad shoofly trestles). For timber piles to be used in permanent construction the pile cutoff must be below the lowest possible ground water level and there must be no exposure to marine borers. Because of their flexibility, low ductility, and difficult cap connections, timber piles are not permitted where seismic considerations are critical.

The maximum allowable design loading (Service Load) for timber piles is 400 kN. Pile information for timber piles should be detailed on the contract plans, similar to other types of driven piling.

Steel HP Piles

Steel HP sections are usually specified when hard driving is anticipated such as where displacement piles cannot penetrate difficult soil layers containing rock, cobbles, gravel, and dense sand. Steel sections are also preferable for longer piles because they are more easily spliced than precast prestressed options. Steel HP piles may not be feasible where highly corrosive soils and/or waters are encountered or where large lateral load resistance is required.

If steel HP piles are allowed as an alternative to a Class pile, the Structure Designer shall provide allowable HP sizes to the Specification Engineer. The HP 360x132 steel pile is usually specified for 900 kN, HP 250x85 for 625 kN and HP 250x62 for 400 kN. The design engineer should note in the Memo to Specification Engineer when other steel sections are acceptable for substitution, and verify with Estimating that a nonstandard HP section is available. Larger pile sections may be required if increased lateral load resistance is needed or hard driving is anticipated. Refer to *BDS* 4.5.6.5.1 for the assumed lateral pile resistance values under Service Loading.

Pile anchors must be designed for the pile's design load in tension. In the case of compression-only piles, a nominal anchor is required. Anchor bars should be epoxy-coated.

Cast-in-Steel-Shell (CISS) Concrete Piles and Steel Pipe Piles

Cast-in-steel-shell concrete piles are driven pipe piles that are filled with cast-in-place reinforced concrete no deeper than the shell tip elevation. CISS piles provide excellent lateral resistance and are a good option under the following conditions: 1) where poor soil conditions exist, such as soft bay mud deposits or loose sands; 2) if liquefaction or scour potential exists that will cause long unsupported pile lengths; or 3) if large lateral soil movements or flows are anticipated from a seismic event.

If composite action is required for flexural capacity, the design engineer must assure that a reliable shear transfer mechanism exists. Welded studs or shear rings may be required, especially for large diameter piles.

CISS piles and steel pipe piles can be driven open ended or closed ended. Caution should be exercised when requiring closed end pipe piles to penetrate very dense granular soils, very hard cohesive soils or soft rock. Generally, pipe piles up to 400 mm in diameter tend to plug during driving while diameters 600 mm and greater tend not to plug. Once plugged, an open-ended pipe behaves like a displacement pile and driving becomes more difficult. When faced with excessive blow counts or high driving stresses, DSF may recommend center relief drilling to achieve the specified tip elevation. When appropriate, DSF will perform a driveability analysis and recommend a pile wall thickness suitable for the expected driving stresses.

The soil plug is left intact at the tip of open-ended CISS piles so that the pile is not undermined during cleaning out. A plug two diameters in length can usually maintain water control, but a seal course may be required for some combinations of high water level and permeable soils.

Non-Destructive Testing of Welds for Steel Piles, Shells, and Casings

Where the tensile or bending strength of steel piles, shells or casings is critical in the design, the special provisions should indicate the zone of the pile that will require nondestructive testing (NDT) on welded splices. Longitudinal and spiral seams in steel pipes are visually inspected at fabrication, but the design engineer may choose to require full NDT on the seam, especially near a welded splice. Because the eventual pile tip elevation is uncertain during driving, the specification of “no-splice zones” in steel piles should be avoided.

The Project Engineer should include the NDT requirements in the Memo to Specification Engineer.

Drilled Piles

Cast-in-Drilled-Hole (CIDH) Concrete Piles

CIDH piles—also known in the industry as drilled shafts or caissons—are a possible solution when driven piles are not suitable, large vertical or lateral resistance is required, or to alleviate constructibility issues. A CIDH pile is more forgiving than a driven pile in terms of noise and vibration, but disposal of hazardous drill spoils may be costly. CIDH piles must be constructible by auger drilling, or pier columns should be used. When battered piles are required, CIDH piles should not be used because of the increased risk of caving and the difficulty of placing concrete in a sloping hole.

CIDH piles rely on friction for most of their capacity. Friction and end bearing are seldom additive because they mobilize peak resistance at different displacements. The situation is worsened in wet conditions, where soft, compressible drill spoils and questionable concrete quality are both possible at the pile tip. DSF recommendations may discard the end bearing component, especially in wet piles.

When ground water is anticipated, CIDH piles must be at least 600 mm in diameter and designed to accommodate the construction techniques associated with drilled piles in wet holes. PVC inspection pipes are installed to permit gamma-gamma and crosshole sonic testing of these CIDH piles. See Attachment 2 for reinforcing steel clearance requirements in conjunction with inspection pipes.

When contemplating CIDH piles in the wet, caution should be exercised in the following cases: 1) single column bent pile extensions lacking redundancy; 2) soft cohesive soils, loose sands, or boulders at the support location (constructibility); and 3) high ground water table that will preclude establishing a differential water pressure head for slurry construction. Driven piles should be considered for these situations. If driven piles cannot be substituted, the designer should anticipate the possibility of defective, *irreparable* piles. In such a case, the Pile Mitigation Plan (Bridge Construction memo 130-9.0) will require replacement or supplemental piles, the location of which should be anticipated in the design phase.

Construction joints should be avoided where ground water is anticipated. If a joint is unavoidable, the plans should show its location and the Special Provisions should describe the required joint preparation. The Memo to Specifications Engineer should highlight the presence of a CIDH splice in the wet.

When Standard Plan CIDH piles are specified for prestressed concrete bridges that utilize a diaphragm type abutment, the detail shown in *Bridge Design Aids* (*BDA*), page 1-4, "DIAPHRAGM ABUTMENT WITH FOOTING" should be used. Displacement due to superstructure prestress shortening creates undesirable stresses in the stiff CIDH piles. See *Memos to Designers* 5-2 for more information.

To ensure constructibility and quality, the length of CIDH piles should be limited to 30 times the pile diameter. When caving conditions exist, the use of a permanent casing should be considered and discussed with DSF. A permanent casing might also be required for CIDH piles close to utilities or traffic (especially in medians) where caving would threaten existing facilities. To prevent binding of the drilling tool, the casing diameter should be at least 200 mm greater than the CIDH pile or rock socket diameter.

To conform to the specifications, the plans must follow a naming convention: steel *casings* are used for constructibility, while driven steel *shells* are used for extra axial capacity. Precise nomenclature is required to direct the contractor to certain bearing specifications. In addition, to verify its geotechnical compressive and tensile capacity, a shell must be driven into place. A casing used for confinement or water control may be driven, drilled, vibrated, or oscillated into place.

The standard specification which allows the contractor to revise specified pile tip elevations is intended for driven piles. Tip revision is generally not allowed for drilled piles. Drilled pile depth is controlled by an engineered length for skin friction, and the drilling process has no inherent measurement (analogous to a blow count) to verify the friction. Building to the specified tip elevation shown on the plans is especially important for single column bents, where a change in the pile's lateral stiffness could affect the dynamics of the entire structure.

The contract items for CIDH piles are as shown in *BDA* Chapter 11. Standard sizes for CIDH augers and steel shells/casings are shown in Attachment 3. These sizes are preferred for CIDH piles with or without a permanent shell or casing. Because the contractor owns and reuses a set of temporary casings, the preferred sizes are required whenever a temporary casing is expected.

Pier Columns

Pier columns are utilized when the presence of rock precludes the use of conventional drilling equipment. Excavation by hand, blasting, and mechanical/chemical splitting are some methods used in hard rock.

Pier column excavation is considerably more expensive than conventional auger drilling and the pay limits must be clearly defined. The pier column cutoff elevation and tip elevation (upper and lower limits of the hard material) should be shown in the Pile Data Table. The pay limits for Structure Excavation (Pier Column) and Structure Concrete (Pier Column) shall be shown on the plans. See *Bridge Design Details* page 7-20 and *BDA* Chapter 11 for details.

Tiedown Anchors

Tiedown anchors can be used where site conditions prevent traditional piles from achieving the necessary tensile capacity. For example, where rock exists close to the ground surface (or scour elevation), piles driven to refusal may be too short to develop the friction that resists uplift. Tiedowns are also effective when combined with spread footings sitting directly on rock, or as part of a seismic retrofit strategy to add uplift capacity to a footing.

Tiedown anchor details typically include prestressing strands or rods grouted into a drilled socket. The final stiffness of the element is dependent on the unbonded length of tendon, and should be accounted for in the design. The prestressing force, if any, increases the tiedown's stiffness by engaging the soil before foundation loads are applied. The lockoff load for fully active systems is 100% of the design load, while passive anchors receive a nominal lockoff, usually 10% of the design load.

Tiedown anchors require no entries in the Pile Data Table. In current practice, the design engineer specifies the unbonded length of tendon, while the contractor calculates the bonded length. Field testing to 125% of the design load verifies the resistance of each tiedown anchor. The test value, "T", must be shown on the plans.

Alternative Piles

The Alternative Pile option is an attempt to take advantage of new pile types that can be used, where appropriate, as alternatives to a State-designed pile. A number of proprietary systems have been approved, including variations on micropiles and grout injection piles. To be approved, each vendor's pile system must go through an extensive review process, including both analysis and full-scale load testing to geotechnical failure.

The design engineer should consult with DSF when a site appears favorable for an Alternative Pile. Alternative Pile designs have been developed in response to site constraints such as low overhead clearance (2 meters minimum), vibration restrictions, and hard-driving soils containing large cobbles. High-capacity micropiles can be successfully installed through an existing pile cap to seismically retrofit a foundation without increasing its size.

When an Alternative Pile is listed in the specifications, the contractor has the option to select an Alternative Pile vendor. The contractor is responsible to prepare pile working drawings and to design the pile to satisfy the demands shown in the Pile Data Table. The pile vendor is required to verify the pile's geotechnical design with a performance test prior to production installation. Proof testing of the production piles is also required.

Load Testing and Dynamic Monitoring of Piles

- DSF may recommend test piles and dynamic monitoring for any of the following conditions:
- At locations where the geotechnical investigation is limited or indicates variable, discontinuous stratigraphy.
- For cast-in-drilled-hole piles located in unproven soil formations.
- To determine whether the specified tip elevation could be revised, including evaluation of pile setup.
- In conjunction with the use of the WAVE equation for bearing analysis.

Dynamic monitoring of driven friction piles correlates the actual capacity determined by static load testing with the expected capacity calculated by construction control methods, such as empirical bearing formulas and Wave Equation Analysis of Driven Piles (WEAP).

Refer to *Standard Plans* B2-9, B2-10 and B2-11 for details and pay limits of Caltrans Standard Plan piles when a pile load test is recommended. A five-pile load test pile group is required when both a tension and compression test is required. A three-pile load test pile group is adequate if only a tension test is required. The Foundation Plan should show pile load test locations, control areas, and the layout of both anchor and load test piles. If possible, all pile load test piles should be incorporated into the permanent structure.

The acceptance criteria, when a pile load test is performed, is a maximum of 13 mm of vertical movement at the top of the pile at the required nominal resistance for both tension and compression.

Corrosion

A site is considered corrosive when any of the following conditions exist: the soil and/or water contains more than 500 ppm of chlorides, more than 2000 ppm of sulfates, has a minimum resistivity of less than 1000 ohm-centimeters, or has a pH of 5.5 or less. The Preliminary Foundation Report or the Final Foundation Report should indicate whether the site is corrosive.

For additional assistance regarding corrosion protection of deep foundations, contact the Corrosion Technology Branch of the Division of Materials Engineering and Testing Services.

Steel Piling

Steel piling may be used in corrosive soil and water environments provided that adequate corrosion mitigation measures are specified. Caltrans typically includes a corrosion allowance (sacrificial metal loss) for steel pile foundations. Other corrosion mitigation measures may include coatings and/or cathodic protection.

Caltrans currently uses the following corrosion rates for steel piling exposed to corrosive soil and water:

Soil Embedded Zone:	0.025 mm per year
Immersed Zone (salt water):	0.100 mm per year
Scour Zone (salt water):	0.125 mm per year
Splash Zone (salt water):	0.150 mm per year

For steel piling driven into undisturbed soil, the region of greatest concern for corrosion is the portion of the pile from the bottom of the pile cap or footing down to 1 meter below the water table. This region of the soil typically has a replenishible source of oxygen needed to sustain corrosion.

The corrosion loss should be doubled for steel H-piling since there are two surfaces on either side of the web and flanges that are exposed to the corrosive soil and/or water. For pipe piles, shells, and casings, the corrosion allowance is only needed for the exterior surface of the pile. The interior surface of the pile (soil plug side) will not be exposed to sufficient oxygen to support significant corrosion.

Concrete Piling

Reinforced concrete piles should be designed in accordance with BDS Article 8.22, "Protection Against Corrosion". Tables 8.22.1 and 8.22.2 of BDS Article 8.22 include specific information regarding concrete cover, use of mineral admixtures, use of a reduced water-to-cementitious material ratio concrete mix, and epoxy coated reinforcing steel for corrosion mitigation against exposure to corrosive soil and/or water.

Memos to Designers 10-05 and 10-06 also provide additional background for the protection of reinforced concrete against corrosion due to chlorides, acids and sulfates, and the use of prefabricated epoxy coated reinforcement for marine environments.

The *Standard Plans* Class 400C, 625C and 900C concrete piles are intended for use in corrosive soils, but they should not be used where the chloride concentration exceeds 10,000 ppm, such as in direct contact with seawater. The Corrosion Technology Branch should be consulted before specifying Class "C" piles at any location.

Tieback and Tiedown Anchors

Both tieback and tiedown anchors are typically specified with corrosion protection. Both types of anchors are sheathed full-length with corrugated plastic and pre-grouted. In addition, the steel in the unbonded area is sheathed with smooth plastic. Both corrugated and smooth plastic can be either polyvinyl chloride or high density polyethylene (HDPE). These anchors systems may also require the use of corrosion inhibiting grease in the unbonded length within the smooth sheathing.

Pile Extensions

The standard pile types (Class 400, 625 and 900) are not intended to be used for pile extensions. When pile extensions are preferable, the pile and the extension shall be designed as a column. Upon request, DSF will provide soil profile data for the designer's analysis of the lateral response of the pile extension. The contract plans should give the option to furnish and drive full-length precast prestressed piles or steel pipe piles. An extended pipe pile should be filled with reinforced concrete from 300 mm below finished grade up to the bent cap. Special seismic detailing may be required to control plastic hinge locations in these pipe pile extensions.

When pile extensions are used for slab bridges, *BDA* Chapter 4 provides appropriate design parameters and a Standard Drawing for three alternative pile details. Compliance with the *Caltrans Seismic Design Criteria* should be verified for the March 1989 details in *BDA*.

Pile extensions may be painted where required for aesthetics and approved by the maintenance engineer.

Seismic Capacity

Uplift Capacity

The details for the standard Class 400 and 400C piles, Class 625 and 625C piles, Class 900 and 900C piles, and 400 mm cast-in-drilled-hole (CIDH) piles have been designed for a nominal axial strength in tension equal to 50 percent of the nominal axial compressive strength. The *Standard Plans*, Sheets B2-3, B2-5, B2-6, and B2-8 show the nominal axial strength for both tension and compression. The demand for uplift resistance at any pile must be limited to the structural capacity of the pile and the pile's connection to the footing.

DSD and DSF must concur that the required tensile axial resistance can be obtained geotechnically. End bearing piles or piles with large end bearing contributions may have limited tensile capacity. When liquefaction or scour is anticipated, the skin friction resistance in all compromised strata is unreliable and should be ignored.

Lateral Capacity

Piles do not normally add significant lateral stiffness to pile caps that are embedded in a competent soil. However, a pile must be capable of conforming to the expected relative displacement between the ground and footing in the event the relative displacement exceeds the pile's elastic displacement capacity. At a minimum, piles must have enough lateral capacity to force plastic hinges into the columns, while still maintaining sufficient axial capacity.

The Class piles detailed in the *Standard Plans* are designed to pin at the pile cap without transferring any appreciable moment. Fixed head piles designed to transfer moment to the pile cap require a case-by-case design that considers the effects on shear, moment, axial load, and stability. The design shall take into consideration lateral pile demands, pile stiffness, and the soil capacity.

Typically, the foundation recommendation for axial capacities does not consider pile penetration depths for lateral loading unless the structure designer makes a request to DSF. When DSF investigates lateral pile resistance, the structure designer must verify that adequate pile penetration is provided for structural stability against scour, liquefaction, or lateral soil flows induced by seismic events. See *Bridge Design Aids* Chapter 12 for guidelines on checking the lateral stability of pile extensions.

Settlement

The standard acceptance criteria for piles require each pile to sustain the nominal resistance with no more than 13 mm vertical displacement. Additional settlement may be considered for Group VII seismic loads if a structural analysis verifies the stability and stress state of the structure above the piles.

Thomas A. Ostrom	**Shannon H. Post**	**Michael A. Barbour**
Bridge Design Office A	Bridge Design Office B	Bridge Design Office C

JPL:kds

ATTACHMENT 1

The Pile Data Table

To ensure contract compliance, the designer must observe several standards when creating the Pile Data Table:

1. If the ENR formula is to be used for pile acceptance criteria, the Design Loading must be shown.
2. The Nominal Resistance* must be shown for *all* piles, regardless of pile type or acceptance criteria. The Standard Specifications allow the Contractor to revise specified tip elevations as long as the required Nominal Resistance is provided and verified through a static load test. The Nominal Resistance is also needed for acceptance by Wave Equation Analysis.
3. The Design Loading shall be rounded up to the nearest 25 kN.
4. The Nominal Resistance shall be rounded up to the nearest 50 kN.
5. When using WSD, the Nominal Resistance is equal to two (2) times the rounded up Design Loading.
6. When ENR is used as the acceptance criteria for LFD, the Design Loading is equal to one half (1/2) the rounded up Nominal Resistance.
7. The Cutoff Elevation is required in the Pile Data Table whenever it cannot be calculated from the bottom of footing elevation and pile head embedment shown on the plans. Pier columns and pile shaft extensions, which have no footings, require a Cutoff Elevation in the Pile Data Table.
8. The Design Tip Elevations for compression, tension, lateral, scour, liquefaction, or a combination of these loads are required on the plans. These elevations express the "intent" of the design and help the field engineer to resolve constructibility and quality issues.
9. The Specified Tip Elevation is the controlling (deepest) value of the Design Tips.
10. When the Specified Tip Elevation is controlled by lateral load, scour, or liquefaction, the Specified Tip Elevation shall not be raised. The static load test to verify nominal resistance cannot duplicate these conditions.

---

*The Nominal Resistance is the analytically estimated load carrying capacity of a foundation calculated using nominal dimensions and material properties, and established soil mechanics principles. Loads as high as the Nominal Resistance are on the verge of failing the soil, but not necessarily the structural member.

ATTACHMENT 1

When requesting specified pile tip elevations, the bridge designer will provide the geotechnical engineer with the following:

- For foundations designed by WSD, provide the Design Loading.
- For foundations designed by LFD, provide the required Nominal Resistance (tension and compression).

The following examples show the required formats for presenting **PILE DATA** on the contract plans. Highlighted in bold font is the information the bridge designer provides to the geotechnical design engineer to determine specified pile tip elevations. In each case the pile loads were determined at the abutments by using WSD and at the bents by using LFD.

- Example #1 – Standard Plan Piles
- Example #2 – Steel HP Piles and CIDH Piles w/ Permanent Steel Casing
- Example #3 – Steel Pipe Piles, Large CISS Piles and Pier Columns
- Example #4 – Large CIDH Piles and Steel Pipe Piles
- Example #5 – Large CIDH Piles w/ Driven Steel Shells

ATTACHMENT 1

Example #1 Standard Plan Piles

A three span overcrossing with single column bents and seat abutments uses Standard Plan piles. Shown in the Table 1 are the actual load demands on the piles based upon WSD and LFD. Standard Plan Class 625 piles will be utilized at the abutments and Class 900 piles at Bent 2 and 400-mm CIDH at Bent 3. Standard Specification 49-1.08 (ENR) will be used as the pile acceptance criteria.

Table 1

Location	WSD Design Loading		LFD Factored Load (Controlling Load Group)	
	Compression	Tension	Compression	Tension
Abut 1	583 kN	0 kN	---	---
Bent 2	---	---	1621 kN (VII)	730 kN (VII)
Bent 3			1604 kN (VII)	739 kN (VII)
Abut 4	567 kN	0 kN	---	---

Table 2 shows the foundation information that the structure designer puts on the contract plans. Shown in **bold font** are the values that the structure designer will provide to the geotechnical design engineer to determine the specified pile tip elevations. Note that when seismic loads (Group VII) control the factored load, the strength reduction factor is $\phi = 1.0$, so that the nominal resistance equals the pile's factored load demand. Strength reduction factors are not applied in WSD, so there is no adjustment based on load group. As with many foundations using large pile groups in competent soil, the lateral load on each pile is low and the associated design tip elevation was not calculated.

Table 2

PILE DATA TABLE

Location	Pile Type	Design Loading	Nominal Resistance		Design Tip Elevations	Specified Tip Elevations
			Compression	Tension		
Abutment 1	Class 625	600 kN	1200 kN	0 kN	47.0(1)	47.0
Bent 2	Class 900	825 kN	1650 kN	750 kN	45.0(1); 55.0(2)	45.0
Bent 3	400mm CIDH (900 kN)	N/A	1650 kN	750 kN	45.0(2); 50.0(1)	45.0
Abutment 4	Class 625	575 kN	1150 kN	0 kN	48.0(1)	48.0

Design tip elevation is controlled by the following demands:
(1) Compression, (2) Tension

ATTACHMENT 1

Example #2 Steel HP Piles and CIDH Piles with Permanent Steel Casing

The following example is for a typical undercrossing with multi-column bents (pinned footings) and seat abutments. Driven non-displacement steel piles will be utilized at the abutments and CIDH piles with permanent steel casing at the bent due to utility conflicts and vibration concerns. The permanent steel casing is for water control only and is not designed to develop any geotechnical resistance. Shown in Table 3 are the actual load demands on the piles based upon WSD and LFD. Standard Specification 49-1.08 (ENR) will be used as the pile acceptance criteria at Abutments 1 and 3.

Table 3

Location	WSD Design Loading		LFD Factored Load (Controlling Load Group)	
	Compression	Tension	Compression	Tension
Abut 1	610 kN	0 kN	---	---
Bent 2	---	---	1403 kN (I)	23 kN (IV)
Abut 3	621 kN	0 kN	---	---

Table 4 shows the foundation information that the structure designer puts on the contract plans. Shown in **bold font** are the values that the structure designer will provide to the geotechnical design engineer to determine the specified pile tip elevations. Note that non-seismic loads control at Bent 2, so the controlling factored load is divided by $\phi = 0.75$ to get the nominal resistance. Also note that since ENR is not used for a drilled pile, no Design Loading is shown at Bent 2.

Table 4

PILE DATA TABLE

Location	Pile Type	Design Loading	Nominal Resistance		Steel Casing Specified Tip Elevation	Design Tip Elevations	Specified Tip Elevations
			Compression	Tension			
Abutment 1	HP250X85	625 kN	1250 kN	0 kN	N/A	32.0(1)	32.0
Bent 2	1.2m CIDH w/permanent steel casing	N/A	1900 kN	50 kN	40.0	25.0(1); 35.0(2)	25.0
Abutment 3	HP 250X85	625 kN	1250 kN	0 kN	N/A	32.0(1)	32.0

Design tip elevation is controlled by the following demands:
(1) Compression, (2) Tension

ATTACHMENT 1

Example # 3 Steel Pipe Piles, Large CISS Piles and Pier Columns

The following example is for a multi-span river crossing. Steel pipe piles will be utilized at the abutments, CISS piles at the river piers, and pier columns at piers away from the river. Standard Specification 49-1.08 (ENR) will be used as the pile acceptance criteria at Abutments 1 and 6. At Piers 2 and 3, "Wave Analysis" will be used for pile acceptance. Shown in Table 5 are the actual load demands on the piles based upon WSD and LFD.

Table 5

Location	WSD Design Loading		LFD Factored Load (Controlling Load Group)	
	Compression	Tension	Compression	Tension
Abut 1	841 kN	0 kN	---	---
Pier 2	---	---	4117 kN (VI)	1783 kN (VII)
Pier 3	---	---	3797 kN (VI)	1677 kN (VII)
Pier 4	---	---	17967 kN (VI)	0 kN
Pier 5	---	---	15728 kN (VI)	0 kN
Abut 6	829 kN	0 kN	---	---

Table 6 shows the foundation information that the structure designer puts on the contract plans. Shown in **bold font** are the values that the structure designer will provide to the geotechnical design engineer to determine the specified pile tip elevations. The design tip elevations controlled by lateral loads (3) are calculated by the structure designer. Note (5) is appropriate at Pier 2 because scour controls the design, and at Pier 3 because lateral load controls. Note (5) is not applied at Piers 4 and 5 because tip revisions are not normally allowed for drilled piles.

Table 6

PILE DATA TABLE

Location	Pile Type	Design Loading	Nominal Resistance		Cut-Off Elevation	Design Tip Elevations	Specified Tip Elevations
			Compression	Tension			
Abut 1	PP 406X12.70	850 kN	1700 kN	0 kN	N/A	-3.0(1)	-3.0
Pier 2	CISS PP1067X15.88	N/A	5500 kN	1800 kN	N/A	-22.0(1,4); -10.0(2,4); -18.0(3,4)	-22.0(5)
Pier 3	CISS PP1067X15.88	N/A	5100 kN	1700 kN	N/A	-20.0(1); -12.0(2); -21.0(3)	-21.0(5)
Pier 4	3.0m Pier Column	N/A	24000 kN	0 kN	10.0	-15.0(1); -12.0(3)	-15.0
Pier 5	3.0m Pier Column	N/A	21000 kN	0 kN	15.0	-9.0(1); -10.0(3)	-10.0
Abut 6	PP 406X12.70	850 kN	1700 kN	0 kN	N/A	-3.0(1)	-3.0

Design tip elevation is controlled by the following demands:
(1) Compression; (2) Tension; (3) Lateral Loads; (4) Scour Potential exist to Elev 5.0 @ Pier 2;
(5) Specified Tip Elevation shall not be raised.

ATTACHMENT 1

Example # 4 Large CIDH Piles and Steel Pipe Piles

The following example is for a seismic retrofit of a three span overcrossing. Large CIDH piles will be utilized at both abutments for lateral restraint. Driven steel pipe piles will be used at Bent 2 due to the presence of high ground water. Due to existing structures near Bent 3, CIDH piles will be used to minimize ground disturbance. During the field study ground water was encountered near Bent 3, so 600mm CIDH piles will be used. Standard Specification 49-1.08 (ENR) will be used as the pile acceptance criteria at Bent 2. Shown in Table 7 are the actual load demands on the piles based upon LFD.

Table 7

Location	WSD Design Loading		LFD Factored Load and (Controlling Load Group)	
	Compression	Tension	Compression	Tension
Abut 1	---	---	---	---
Bent 2	---	---	720 kN (V)	305 kN (V)
			880 kN (VII)	445 kN (VII)
Bent 3	---	---	1291 kN (VII)	1270 kN (VII)
Abut 4	---	---	---	---

Table 8 shows the foundation information that the structure designer puts on the contract plans. Shown in **bold font** are the values that the structure designer will provide to the geotechnical design engineer to determine the specified pile tip elevations.

At Bent 2, the nominal resistance in compression is controlled by Group V, even though the Group VII factored load is larger. The strength reduction factor makes the difference: 720/0.75=960 kN is greater than 880/1.0=880 kN; round 960 kN up to 1000 kN for the table. In tension, the seismic load combination controls nominal resistance.

The Cut-Off Elevation is shown at the abutments because the CIDH piles in this design extend all the way up to the superstructure, and there is no bottom of footing elevation to define the cut-off.

Table 8

PILE DATA TABLE

Location	Pile Type	Design Loading	Nominal Resistance		Cut-Off Elevation	Design Tip Elevations	Specified Tip Elevations
			Compression	Tension			
Abut 1	1.8m CIDH	N/A	**0 kN**	**0 kN**	65.0	**50.0(3)**	50.0
Bent 2	PP406X12.70	500 kN	**1000 kN**	**450 kN**	N/A	**45.0(1,4); 41.0(2,4)**	41.0(5)
Bent 3	600mm CIDH	N/A	**1300 kN**	**1300 kN**	N/A	**42.0(1,4); 38.0(2,4)**	38.0
Abut 4	1.8m CIDH	N/A	**0 kN**	**0 kN**	60.0	**44.0(3)**	44.0

Design tip elevations are controlled by the following demands:
(1) Compression; (2) Tension; (3) Lateral Loads; (4) Liquefaction potential exists from Elev. 50 to 55. (5) Specified Tip Elevation shall not be raised.

Example # 5 Large CIDH Piles with Driven Steel Shell

The following example is that of the seismic retrofit of a major river crossing where large CIDH piles are recommended with permanent, driven steel shells. The shell is required to facilitate construction and will be required to develop a portion of the required nominal resistance. The shell is to be installed by driven methods only and "Wave Analysis" will be used for pile acceptance. Shown in the Table 9 are the actual load demands on the piles based upon LFD.

Table 9

Location	LFD Factored Load (Controlling Load Group)	
	Compression	Tension
Pier 5	4961 kN (VII)	2193 kN (VII)

Shown in Table 10 is the foundation information that the structure designer shows on the contract plans. Shown in **bold font** are the values that the structure designer will provide to the geotechnical design engineer to determine the specified pile tip elevations. The geotechnical design engineer will furnish the Nominal Resistance for the driven shell.

Table 10

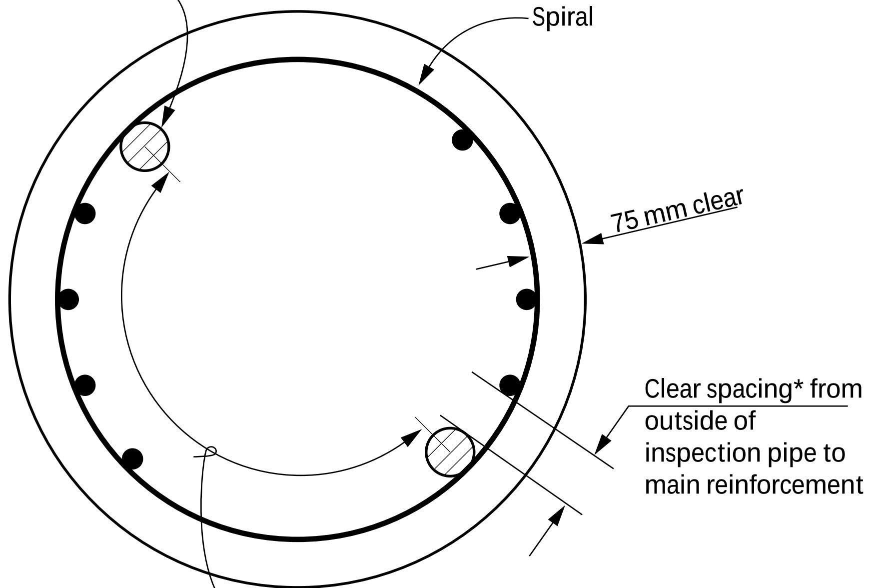
PILE DATA TABLE

Location	Pile Type	Design Loading	Nominal Resistance		Nominal Resistance (Driven Steel Shell)		Specified Tip Elevation (Shell)	Design Tip Elevations	Specified Tip Elevations (CIDH)
			Compression	Tension	Compression	Tension			
Pier 5	1.6m CIDH w/PP1800X19.05 Driven Steel Shell	N/A	5000 kN	2200 kN	2600 kN	1100 kN	-50.0	-55.0(1) -65.0(2)	-65.0

Design tip elevations are controlled by the following demands:
(1) Compression; (2) Tension

ATTACHMENT 2

CIDH Inspection Pipes



* 50 mm clear for #25 and smaller not bundled main reinforcement; 75 mm clear for other reinforcing configurations.

Notes to Designers:

1. Inspection tubes shall not be shown on the plans (placement is covered by the contract specifications).
2. Inspection tubes are on the perimeter of the pile only and are not placed in the middle of the pile, even for large diameter piles.

ATTACHMENT 3

Standard Metric Sizes

Design engineers should show on the plans standard metric sizes for CIDH concrete piling. The actual auger used for the work may be an existing imperial size:

Size Shown on Plans	Actual Imperial Tool Used
350 mm	14"
400 mm	16"
450 mm	18"
600 mm	24"
750 mm	30"
1.0 m	42"
1.2 m	48"
1.5 m	60"
1.8 m	72"
2.1 m	84"
2.4 m	96"
3.0 m	120"
3.6 m	144"
4.0 m	156"

ATTACHMENT 3

Design engineers should show on the plans the industry's standard sizes for pipe piles, casings, and shells:

Size Shown on Plans	Equivalent Imperial Size
PP360 x 4.55	NPS 14 x 0.179
PP 360 x 6.35	NPS 14 x 0.250
PP 360 x 9.53	NPS 14 x 0.375
PP 360 x 11.12	NPS 14 x 0.438
PP 406 x 12.70	NPS 16 x 0.500
PP 460 x T	NPS 18 x T"
PP 508 x T	NPS 20 x T"
PP 559 x T	NPS 22 x T"
PP 610 x T	NPS 24 x T"
PP 660 x T	NPS 26 x T"
PP 711 x T	NPS 28 x T"
PP 762 x T	NPS 30 x T"
PP 813 x T	NPS 32 x T"
PP 864 x T	NPS 34 x T"
PP 914 x T	NPS 36 x T"
PP 965 x T	NPS 38 x T"
PP 1016 x T	NPS 40 x T"
PP 1067 x T	NPS 42 x T"
PP 1118 x T	NPS 44 x T"
PP 1219 x T	NPS 48 x T"
PP 1524 x T	NPS 60 x T"

The NPS diameter is the outside diameter of the pipe. The thickness in mm (T) should be an exact conversion of one of the standard imperial thicknesses in inches (T"). Pile diameters greater than 1524 mm are nonstandard and any combination of metric diameter and thickness can be fabricated.